# STUDY OF SUBSTITUTE FRAME METHOD OF ANALYSIS FOR LATERAL LOADING CONDITIONS

A THESIS SUBMITTED IN PARTIAL FULFILLMENT FOR THE DEGREE OF

**BACHELOR OF TECHNOLOGY**

**IN**

**CIVIL ENGINEERING**

**BY**

**ABHISHEK MEHTA**

**ROLL NO. – 107CE032**

Department of Civil Engineering

National Institute of Technology, Rourkela

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**Under the guidance of-**

**Prof. A. K. Sahoo**

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## CERTIFICATE

This is to certify that this report entitled, "STUDY OF SUBSTITUTE FRAME METHOD OF ANALYSIS FOR LATERAL LOADING CONDITIONS" submitted by Abhishek Mehta in partial fulfillments for the requirements for the award of Bachelor of Technology Degree in Civil Engineering at National Institute of Technology, Rourkela (Deemed University) is an authentic work carried out by him under my supervision and guidance.

To the best of my knowledge, the matter embodied in this report has not been submitted to any other University / Institute for the award of any Degree or Diploma

Date:

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# ABSTRACT

Analysis of multi-storey building frames involves lot of complications and tedious calculations by conventional methods. To carry out such analysis is a time consuming task. Substitute frame method for analysis can be handy in approximate and quick analysis so as to get the estimates ready and participate in the bidding process. Till date, this method has been applied only for vertical loading conditions. In this work, the applicability and effectiveness of this method has been checked under lateral loading conditions.

# CHAPTER 1

## HISTORY

# HISTORY

## STRUCTURAL ANALYSIS

A structure refers to a system of two or more connected parts use to support a load. It is an assemblage of two or more basic components connected to each other so that they serve the user and carry the loads developing due to the self and super-imposed loads safely without causing any serviceability failure. Once a preliminary design of a structure is fixed, the structure then must be *analyzed* to make sure that it has its required strength and rigidity. To analyze a structure a structure correctly, certain idealizations are to be made as to how the members are supported and connected together. The loadings are supposed to be taken from respective design codes and local specifications, if any. The forces in the members and the displacements of the joints are found using the theory of structural analysis.

The whole structural system and its loading conditions might be of complex nature so to make the analysis simpler, we use certain simplifying assumptions related to the quality of material, member geometry, nature of applied loads, their distribution, the type of connections at the joints and the support conditions. This shall help making the process of structural analysis simpler to quite an extent.

## Methods of structural analysis

When the number of unknown reactions or the number of internal forces exceeds the number of equilibrium equations available for the purpose of analysis, the structure is called as a *statically indeterminate structure.* Most of the structures designed today are statically indeterminate. This indeterminacy may develop as a result of added supports or extra members, or by the general form of the structure.

While analyzing any indeterminate structure, it is essential to satisfy equilibrium, compatibility, and force-displacement requisites for the structure. When the reactive forces hold the structure at rest, equilibrium is satisfied and compatibility is said to be satisfied when various segments of a structure fit together without intentional breaks or overlaps.

Two fundamental methods to analyze the statically indeterminate structures are discussed below.

**Force methods-**

Originally developed by James Clerk Maxwell in 1864, later developed by Otto Mohr and Heinrich Muller-Breslau, the force method was one of the first methods available for analysis of statically indeterminate structures. As compatibility is the basis for this method, it is sometimes also called as *compatibility method* or the *method of consistent displacements.* In this method, equations are formed that satisfy the compatibility and force-displacement requirements for the given structure in order to determine the redundant forces. Once these forces are determined, the remaining reactive forces on the given structure are found out by satisfying the equilibrium requirements.

**Displacement methods-**

The displacement method works the opposite way. In these methods, we first write load-displacement relations for the members of the structure and then satisfy the equilibrium requirements for the same. In here, the unknowns in the equations are displacements.

Unknown displacements are written in terms of the loads (i.e. forces) by using the load-displacement relations and then these equations are solved to determine the displacements. As the displacements are determined, the loads are found out from the compatibility and load- displacement equations. Some classical techniques used to apply the displacement method are discussed.

**Slope deflection method-**

This method was first devised by Heinrich Manderla and Otto Mohr to study the secondary stresses in trusses and was further developed by G. A. Maney extend its application to analyze indeterminate beams and framed structures. The basic assumption of this method is to consider the deformations caused only by bending moments. It's assumed that the effects of shear force or axial force deformations are negligible in indeterminate beams or frames.

The fundamental slope-deflection equation expresses the moment at the end of a member as the superposition of the end moments caused due to the external loads on the member, while the ends being assumed as restrained, and the end moments caused by the displacements and actual end rotations. A structure comprises of several members, slope-deflection equations are applied to each of the member. Using appropriate equations of equilibrium for the joints along with the slope-deflection equations of each member we can obtain a set of simultaneous equations with unknowns as the displacements. Once we get the values of these unknowns i.e. the displacements we can easily determine the end moments using the slope-deflection equations.

**Moment distribution method-**

This method of analyzing beams and multi-storey frames using moment distribution was introduced by Prof. Hardy Cross in 1930, and is also sometimes referred to as Hardy Cross method. It is an iterative method in which one goes on carrying on the cycle to reach to a desired degree of accuracy. To start off with this method, initially all the joints are temporarily restrained against rotation and fixed end moments for all the members are written down. Each joint is then released one by one in succession and the unbalanced moment is distributed to the ends of the members, meeting at the same joint, in the ratio of their distribution factors. These distributed moments are then carried over to the far ends of the joints. Again the joint is temporarily restrained before moving on to the next joint. Same set of operations are performed at each joints till all the joints are completed and the results obtained are up to desired accuracy. The method does not involve solving a number of simultaneous equations, which may get quite complicated while applying large structures, and is therefore preferred over the slope-deflection method.

**Kani's method-**

This method was first developed by Prof. Gasper Kani of Germany in the year 1947. The method is named after him. This is an indirect extension of slope deflection method. This is an efficient method due to simplicity of moment distribution. The method offers an iterative scheme for applying slope deflection method of structural analysis. Whereas the moment distribution method reduces the number of linear simultaneous equations and such equations needed are equal to the number of translator displacements, the number of equations needed is zero in case of the Kani's method.

This method may be considered as a further simplification of moment distribution method wherein the problems involving sway were attempted in a tabular form thrice (for double story frames) and two shear coefficients had to be determined which when inserted in end moments gave us the final end moments. All this effort can be cut short very considerably by using this method.

→ Frame analysis is carried out by solving the slope−deflection equations by successive approximations. Useful in case of side sway as well.

→ Operation is simple, as it is carried out in a specific direction. If some error is committed, it will be eliminated in subsequent cycles if the restraining moments and distribution factors have been determined correctly.

**RULES FOR CALCULATING ROTATION CONTRIBUTIONS-**

**Case-1: Without side sway**

Definition: "Restrained moment at a joint is the algebraic sum of FE.M's of different members meeting at that joint."

**1.** Sum of the restrained moment of a joint and all rotation contributions of the far ends of members meeting at that joint is multiplied by respective rotation factors to get the required near end rotation contribution. For the first cycle when far end contributions are not known, they may be taken as zero (1st approximation).

**2.** By repeated application of this calculation procedure and proceeding from joint to joint in an arbitrary sequence but in a specific direction, all rotation contributions are known. The process is usually stopped when end moment values converge.

**Case 2: With side sway (Joint translations)**

In this case in addition to rotation contribution, linear displacement contributions (Sway contributions) of columns of a particular storey are calculated after every cycle as follows: For the first cycle,

**(A)→ Linear displacement contribution (LDC) of a column** = LDF of a particular column of a storey × ( storey moment + contributions at the ends of columns of that storey)

Linear displacement factor (LDF) for columns of a storey = $-3/2$

Linear displacement factor of a column = $-3/2(k/\Sigma k)$

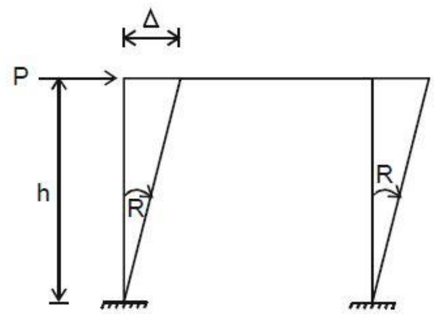
Where k =stiffness of the column being considered and

$\Sigma k$ = sum of stiffness of all columns of that storey

**(B) →Storey moment** = Storey shear x storey height/3

**(C) → Storey shear**: It is considered as reaction of column at horizontal beam / slab levels due to lateral loads by considering the columns of each storey as simply supported beams in vertical direction. "If applied load gives + R value (according to sign conversion of slope deflection method), storey shear is +ve or vice versa."

Consider a general sway case,




SIGN CONVENTION ON MOMENTS: – Counter-clockwise moments are positive and

Clockwise rotations are positive.

For first cycle with side sway,

**(D)**Near end contribution of various = Rotation contribution factor ×
members meeting at that joint (Restrained moment + far end contributions)

**Linear displacement contributions are calculated after the end of each cycle for the columns only.**

For second and subsequent cycles,

**(E)** → Near end contributions of various = Rotation contribution factor ×
members meeting at a joint (Restrained moment + far end contributions+ Linear displacement contribution of columns of different storeys meeting at that joint)

**Rules for the Calculation of final end moments (side sway cases)**

**(F)** →For beams, End moment = FEM + 2 near end contribution + Far end contributions

**(G)** →For columns, End moment = FEM + 2 near end contribution + Far end contribution + linear displacement contribution of that column for the last cycle

**Advantages of Kani's method:**

- All the computations are carried out in a single line diagram of the structure.
- The effects of joint rotations and sway are considered in each cycle of iteration. Henceforth, no need to derive and solve the simultaneous equations. This method thus becomes very effective and easy to use especially in case of multistory building frames.
- The method is self correcting, that is, the error, if any, in a cycle is corrected automatically in the subsequent cycles. The checking is easier as only the last cycle is required to be checked.
- The convergence is generally fast. It leads to the solutions in just a few cycles of iterations.

# CHAPTER 2

## INTRODUCTION

# Introduction

## Idea

Structural analysis is the backbone of civil engineering. During recent years, there has been a growing emphasis on using computer aided softwares and tools to analyze the structures. There has also been advancement in finite element analysis of structures using Finite Element Analysis methods or matrix analysis. These developments are most welcome, as they relieve the engineer of the often lengthy calculations and procedures required to be followed while large or complicated structures are analyzed using classical methods. But not all the time such detailed analysis are necessary to be performed i.e. sometimes, just approximate analysis could suffice our requirements as in case of preparing the rough estimates and participating in the bidding process for a tender. It may even happen that sometimes the analysis software or tool is not available at hand? Or the worst case, the computer itself is not available?? Then in such cases, accurate analysis of such large and complicated structures involving so many calculations is almost impossible.

Now-a-days, high rise buildings and multi-bay-multi-storey buildings are very common in metropolitan cities. The analysis of frames of multi-storeyed buildings proves to be rather cumbersome as the frames have a large number of joints which are free to move. Even if the commonly used Moment distribution method is applied to all the joints, the work involved shall be tremendous. However, with certain assumptions, applying the substitute analysis methods like substitute frame method, portal method, cantilever method or factor method, the structures can be analyzed approximately.

**Substitute frame method**

By considering any floor of the frame called substitute frame, the moments can be calculated and results can be obtained in good agreement with the results from rigorous analysis. The moments carried from floor to floor through columns are very small as compared to the beam moments; therefore, the moments in one floor have negligible effect on the moments on the floors above and below. Therefore, in this method, the analysis of the multi-storeyed frames is carried out by taking one floor at a time. Each floor is taken with columns above and below fixed at far ends, and the moments and shears are calculated in beams and columns.

The method is very effective in analyzing any framed structure under vertical loadings. This work is focused to check its applicability and efficacy under the lateral loading conditions

**Objectives**

- To manually analyze the problem frame, using Kani's method under both vertical and lateral loading conditions.
- To perform the same analysis using standard analysis software Staad.Pro
- Perform substitute frame analysis for both the loading cases
- Compare the accuracy of the substitute frame analysis with manual and Staad.Pro analysis and check its validity in lateral loading cases.
- Optimize the substitute frame method to further lessen the calculations so as to get the final results within permissible limit of errors.

**METHODOLOGY ADOPTED**

- A 4-storey-4-bay unsymmetrical frame structure was set as a problem frame.
- Kani's analysis was performed for vertical loading conditions.
- Staad Pro. analysis was performed to verify the Kani's analysis.
- Same problem frame was analyzed using substitute frame method for vertical loads.
- Approximate substitute frame analysis results were then compared with those found by the accurate Kani's analysis and corresponding percentage deviations were determined.
- Again Kani's analysis was performed for lateral loading conditions; only for the wind loads no seismic forces were considered.
- Substitute frame analysis was performed using some assumptions, then again the results were compared with the Kani's analysis.

# CHAPTER 3

## ANALYSIS UNDER VERTICAL LOADS

**Kani's Analysis (non-sway case)**

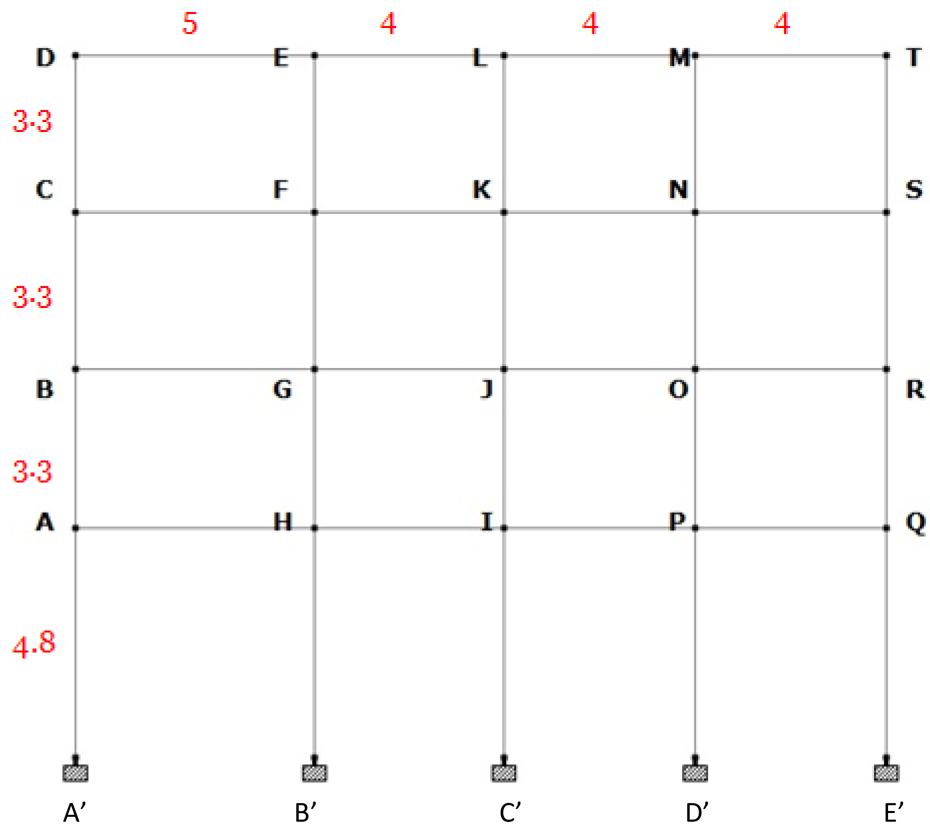



Figure 3.1

**Assumptions-**

Slab thickness = 0.12 m

Floor finish thickness = 0.05 m

Beam section = 0.3m×0.4m

Column section = 0.3m × 0.45m

Density of concrete used = 25 kN/m$^3$

Live load for residential building = 2 kN/m$^2$

Clockwise moment positive and vice-versa.

**Loading-**

Slab dead load = $0.12 \times 1 \times 25 = 3\ kN/m^2$

Floor finish load = $1.25\ kN/m^2$

Live load = $2\ kN/m^2$ ----------------------------------- (assuming a residential building)

Beam self weight = $0.3 \times 0.4 \times 25 = 3\ kN/m$

Total vertical load per meter length of beam = $(3+1.25+2) \times 4 + 3 = 28\ kN/m$

**Fixed end moments induced-**

Mfah = Mfbg = Mfcf = Mfde = $-\frac{28\times5\times5}{12} = -58.3$ kNm

Mfhi= Mfgj=Mffk= Mfel=Mflm=Mfkn=Mfjo=Mfip=Mfpq=Mfor=Mfns=Mfnt =$-\frac{28\times4\times4}{12}$

$= -37.3$ kNm

Mfha = Mfgb = Mffc = Mfed = $\frac{28\times5\times5}{12} = 58.3$ kNm

Mfih= Mfjg=Mfkf= Mfle=Mfml=Mfnk=Mfoj=Mfpi=Mfqp=Mfro=Mfsn=Mftn = $\frac{28\times4\times4}{12}$

$=37.3$ kNm

| SPAN | RELATIVE MOMENT OF INERTIA ($I_{REL}$) | LENGTH (L) in meters | $I_{REL}/L$ | RELATIVE STIFFNESS ($K_{REL}$) |
|---|---|---|---|---|
| AA’ | 1.42 | 4.8 | 1.42/4.8 | 93.5 |
| AB | 1.42 | 3.3 | 1.42/3.3 | 136 |
| AH | 1 | 5 | 1/5 | 63.2 |
| HI | 1 | 4 | 1/4 | 79 |
| HG | 1.42 | 3.3 | 1.42/3.3 | 136 |
| HB’ | 1.42 | 4.8 | 1.42/4.8 | 93.5 |
| HA | 1 | 5 | 1/5 | 63.2 |
| IJ/PO | 1.42 | 3.3 | 1.42/3.3 | 136 |
| IH/PI | 1 | 4 | 1/4 | 79 |
| IC’/PD’ | 1.42 | 4.8 | 1.42/4.8 | 93.5 |
| IP/PQ | 1 | 4 | 1/4 | 79 |
| QP | 1 | 4 | 1/4 | 79 |
| QR | 1.42 | 3.3 | 1.42/3.3 | 136 |
| QE’ | 1.42 | 4.8 | 1.42/4.8 | 93.5 |
| BG/CF | 1 | 5 | 1/5 | 63.2 |
| BC/CD | 1.42 | 3.3 | 1.42/3.3 | 136 |
| BA/CB | 1.42 | 3.3 | 1.42/3.3 | 136 |
| GF/FE | 1.42 | 3.3 | 1.42/3.3 | 136 |
| GB/FC | 1 | 5 | 1/5 | 63.2 |

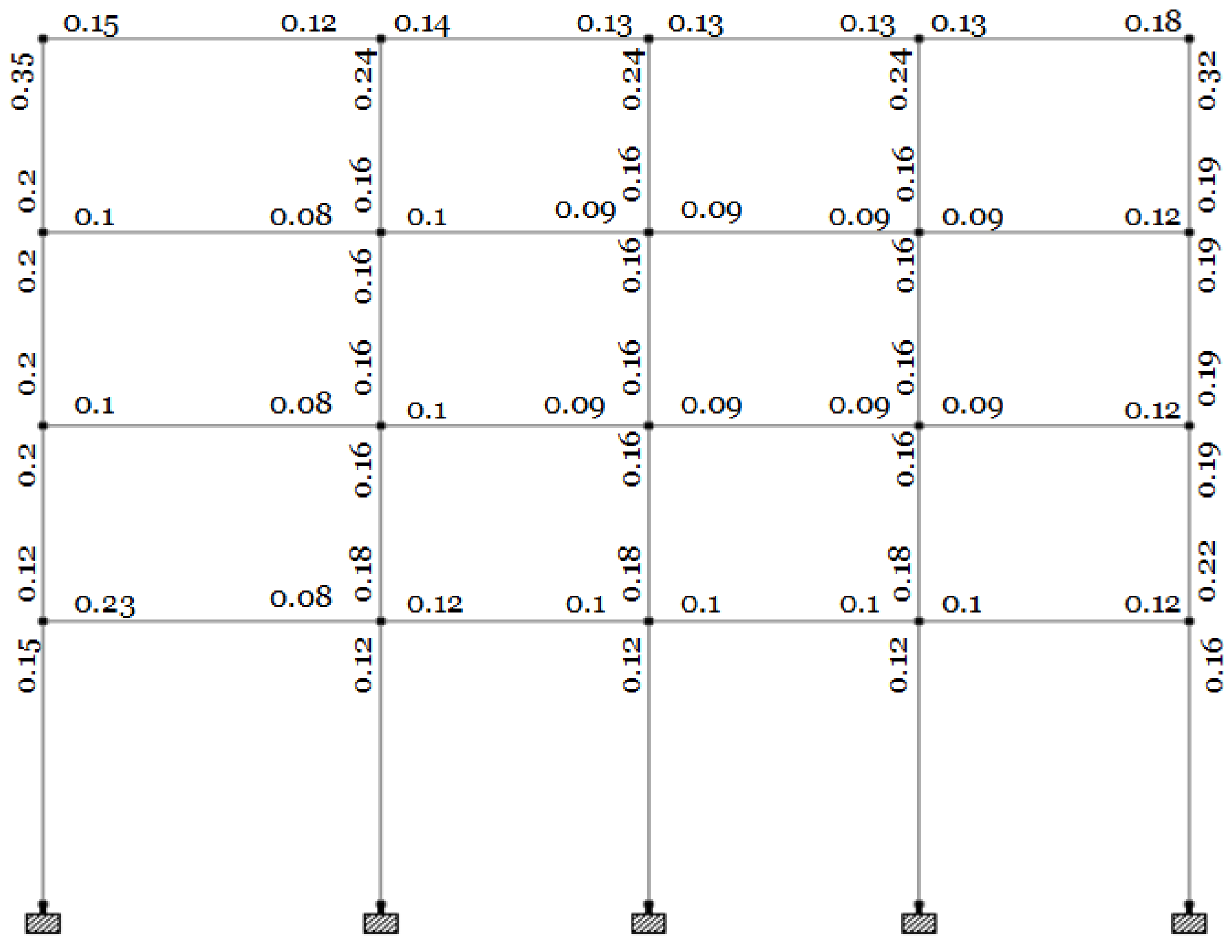
| | | | | |
|---|---|---|---|---|
| GH/FG | 1.42 | 3.3 | 1.42/3.3 | 136 |
| GJ/FK | 1 | 4 | 1/4 | 79 |
| JK/ON/KL/NM | 1.42 | 3.3 | 1.42/3.3 | 136 |
| JG/OJ/KF/NK | 1 | 4 | 1/4 | 79 |
| JI/OP/KJ/NO | 1.42 | 3.3 | 1.42/3.33 | 136 |
| JO/OR/KN/NS | 1 | 4 | 1/4 | 79 |
| RS/ST | 1.42 | 3.3 | 1.42/3.3 | 136 |
| RO/SN | 1 | 4 | 1/4 | 79 |
| RQ/SR | 1.42 | 3.3 | 1.42/3.3 | 136 |
| DC | 1.42 | 3.3 | 1.42/3.3 | 136 |
| DE | 1 | 5 | 1/5 | 63.2 |
| ED | 1 | 5 | 1/5 | 63.2 |
| EF | 1.42 | 3.3 | 1.42/3.3 | 136 |
| EL | 1 | 4 | 1/4 | 79 |
| LE/ML | 1 | 4 | 1/4 | 79 |
| LK/MN | 1.42 | 3.3 | 1.42/3.3 | 136 |
| LM/MT | 1 | 4 | 1/4 | 79 |
| TM | 1 | 4 | 1/4 | 79 |
| TS | 1.42 | 3.3 | 1.42/3.3 | 136 |

All the rotation contribution factors at each joint were calculated,

RCF = -1/2 × k/ Σk

k = relative stiffness of the member

Σk = sum of the relative stiffness of members meeting at the joint




All the restrained moments at every joint were calculated,

Restrained moment = Algebraic sum of $FEM_S$ induced in the members meeting at that joint

Rotation contribution (Near end contribution) = RCF × (RM + Far end contributions)

Numbers of cycles were performed till the Near End Contributions (NEC) converged.

Note – All the RCF values are negative.

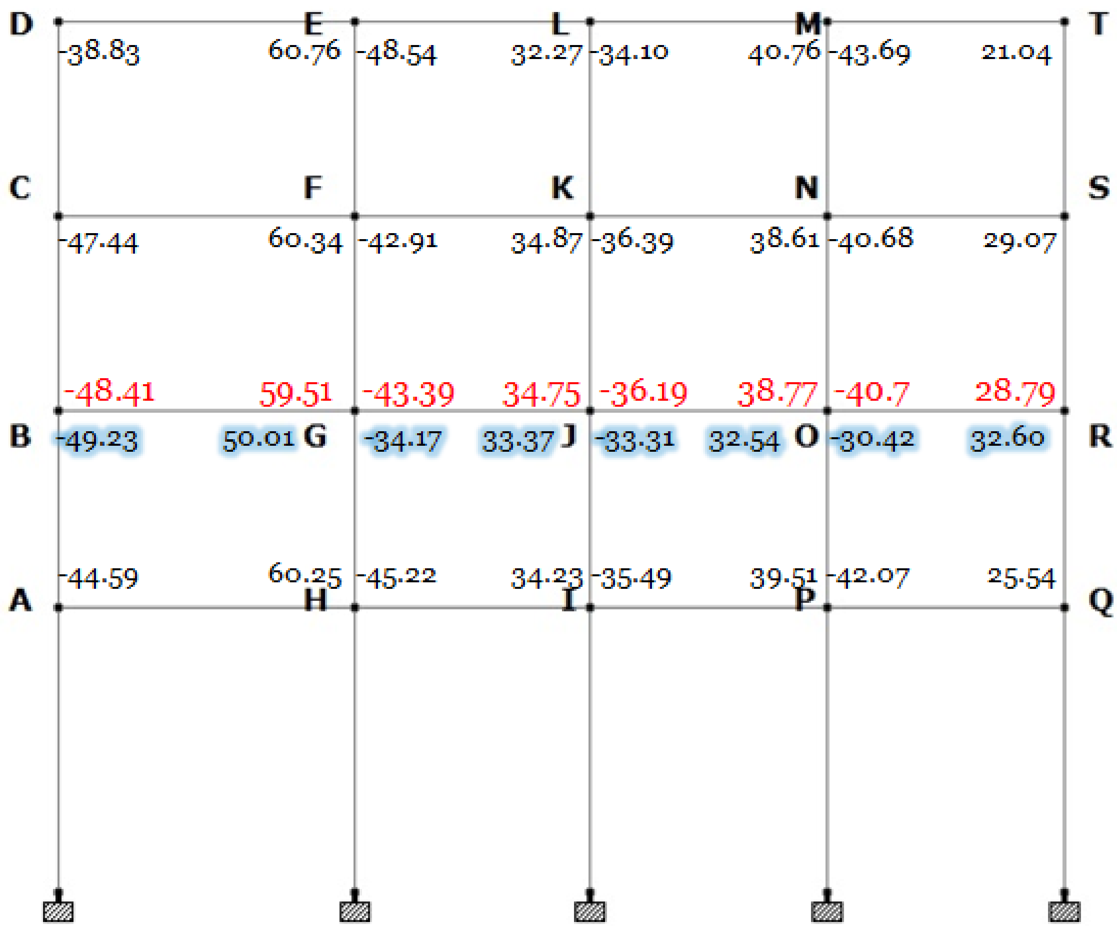
**For beams –**

End moments = FEM + 2× NEC + FEC

**For columns –**

End moments = FEM + 2× NEC + FEC + LDC

LDC – Linear displacement contribution =0 (since it's a non sway case)




Final calculated end moments for beams are shown in the above frame at each joint.

The values for the end moments for the 2nd floor using STAAD. Pro are highlighted in blue.

This was just to verify Kani's analysis results, which were found to be approximately same.

**SUBSTITUTE FRAME METHOD**

- 2nd floor of the frame was considered.
- Column ends of the floor on both sides were assumed to be fixed.
- Distribution factors depending upon the member stiffness were calculated for each member.
- Total $FEM_S$ and Dead load $FEM_S$ were calculated with all spans loaded.
- Distribution of moments was performed to get the final end moments.

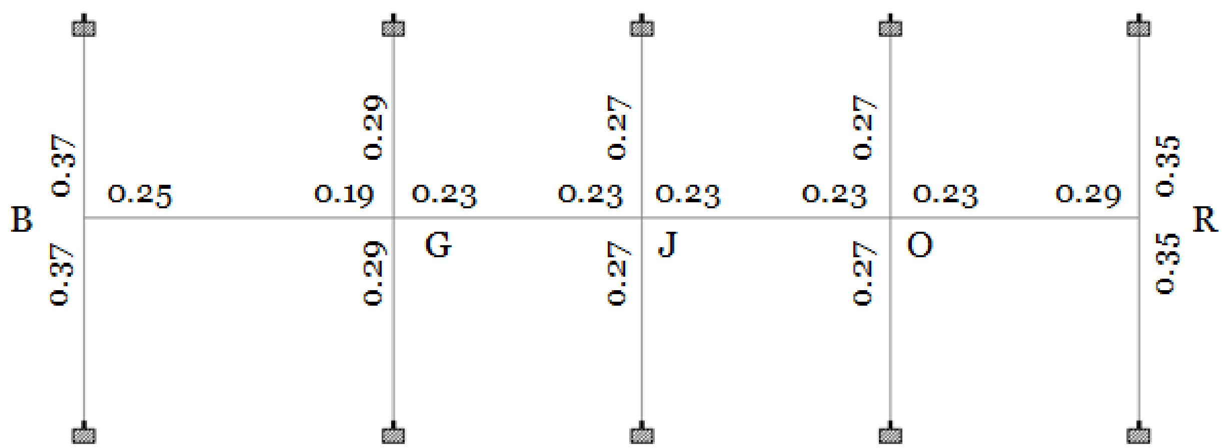
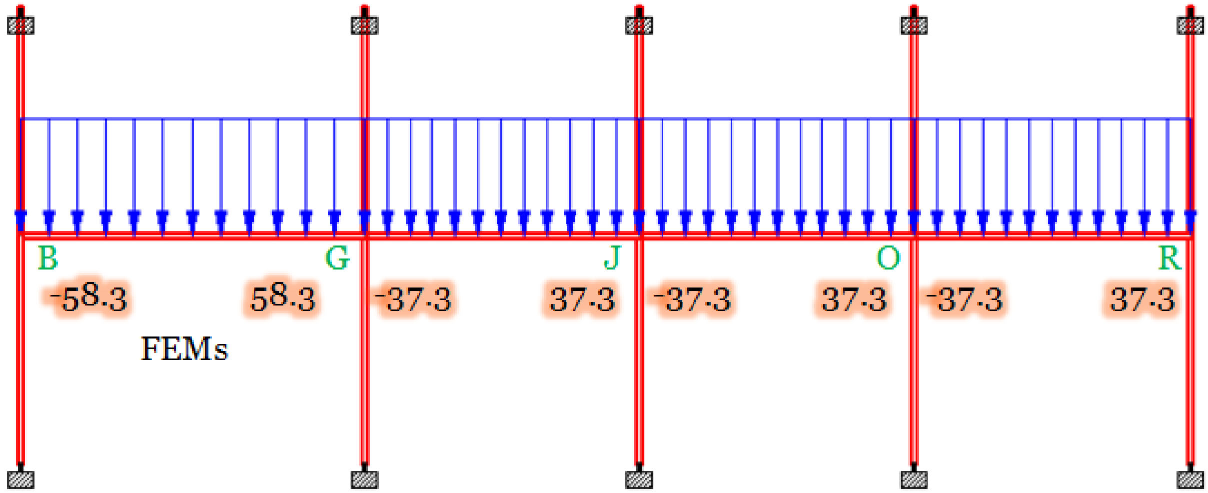



Figure 2

| SPAN | DEAD LOAD FEM | TOTAL FEM |
|---|---|---|
| BG/GB | 41.6 | 58.3 |
| GJ/JG | 26.6 | 37.3 |
| JO/OJ | 26.6 | 37.3 |
| OR/RO | 26.6 | 37.3 |




| JOINTS | B | G | | J | | O | | R |
|---|---|---|---|---|---|---|---|---|
| MEMBERS | BG | GB | GJ | JG | JO | OJ | OR | RO |
| DISTRIBUTION FACTORS | 0.25 | 0.19 | 0.23 | 0.23 | 0.23 | 0.23 | 0.23 | 0.29 |
| FEMS | -58.3 | 58.3 | -37.3 | 37.3 | -37.3 | 37.3 | -37.3 | 37.3 |
| DISTRIBUTION | 14.57 | -3.99 | -4.83 | 0 | 0 | 0 | 0 | -10.81 |
| CARRY OVER | -1.99 | 7.28 | 0 | -2.41 | 0 | 0 | -5.4 | 0 |
| DISTRIBUTION | 0.49 | -1.38 | -1.07 | 0.55 | 0.55 | 1.24 | 1.24 | 0 |
| CARRY OVER | -0.69 | 0.24 | 0.27 | -0.83 | 0.62 | 0.27 | 0 | 0.62 |
| DISTRIBUTION | 0.17 | -0.09 | -0.11 | 0.04 | 0.04 | -0.06 | -0.06 | -0.17 |
| TOTAL MOMENT | -45.75 | 60.39 | -43.07 | 34.68 | -36.12 | 38.75 | -41.55 | 26.97 |

| Members | STAAD. Pro End Moments (kN) | Kani's Method End Moments(kN) | Substitute frame Method End Moments (kN) | Kani's Method Vs S/F Method |
|---|---|---|---|---|
| **BG** | -49.23 | -48.41 | -45.75 | 5.49 % |
| **GB** | 50.01 | 59.51 | 60.39 | 1.47 % |
| **GJ** | -34.17 | -43.39 | -43.07 | 0.7 % |
| **JG** | 33.37 | 34.75 | 34.68 | 0.2 % |
| **JO** | -33.31 | -36.19 | -36.12 | 0.2 % |
| **OJ** | 32.54 | 38.77 | 38.75 | 0 |
| **OR** | 30.42 | -40.70 | -41.55 | 2.08 % |
| **RO** | 32.60 | 28.79 | 26.97 | 6 % |

The inference made from the graph is that in case of vertical loading, the difference between the Kani's analysis and substitute frame method (S/F method) is very less.

# CHAPTER 4

## ANALYSIS UNDER LATERAL LOAD

## (WIND LOAD)

**Kani's Analysis (sway case)**

- Design wind pressure for the region was assumed to be 1.5 kN/$m^2$
- Along with the vertical loads, the frame was assumed to be resisting the wind pressure for a length of 4m i.e. the spacing between the frames was assumed to be 4m.
- Rotation contribution factors were same as calculated in the vertical loading case.
- Restrained moments were calculated again.
- Same steps were then followed as in case of vertical loading analysis, repeating the cycles till the values of Near end contributions converged.

**For 1st cycle**, Near end contribution (NEC) = RCF × (restrained moment + FECs)

**From 2nd cycle onwards**, NEC = RCF × (restrained moment + FECs + LDCs of columns of different storeys meeting at that joint)

where linear displacement contribution (LDC) = LDF × (storey moment + NEC at the end of columns of that storey)

Linear displacement factor (LDF) = -3/2 × k/ Σk

k = relative stiffness of column

Σk = sum of relative stiffness of columns of that storey

Storey moment = storey shear × storey height/ 3

| FLOOR | LDF | STOREY SHEAR (kN) | STOREY MOMENT (kNm) |
|---|---|---|---|
| 1st | -0.3 | -24.3 | -38.8 |
| 2nd | -0.3 | -19.8 | -21.7 |
| 3rd | -0.3 | -19.8 | -21.7 |
| 4th | -0.3 | -9.9 | -10.9 |

Final end moments of our desired floor were as given below,

Mbg = -54.84 kNm

Mgb = 56.67 kNm

Mgj = -43.94

Mjg = 32.66 kNm

Mjo = -39.93 kNm

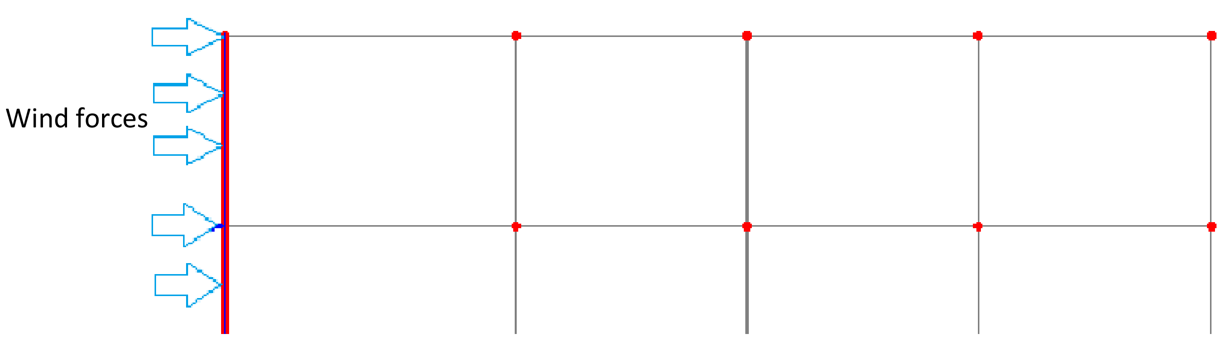
Moj = 34.68 kNm

Mor = -46.09 kNm

Mro = 31.6 kNm

**Substitute frame method**

- We assume that the wind load moments are resisted by the resisting moments arising at the joints.
- These resisting moments are contributed by the members meeting at the joint, including beams, in proportion to their distribution factors.
- The wind load moments are calculated by considering a section in the floor above the floor under consideration i.e. above the 2nd floor (BGJOR).
- The shear force (maximum) is found out at that section which ultimately induces the resisting moment at the joint.
- It is assumed that the interior columns resist double the shear force than that resisted by the exterior ones.
- The unbalanced joint moment is then distributed to get the final moments in each member.
- This final moment is then superimposed with the final moment due to vertical loading to get the combined final moments.



Total shear force induced due to wind loads = 1.5 × 4 (3.3 + 3.3/2)

= 29.7 kN

Force resisted by each exterior column = 29.7/8 = 3.71 kN

Force resisted by each interior column = 2 × 3.71 = 7.4 kN

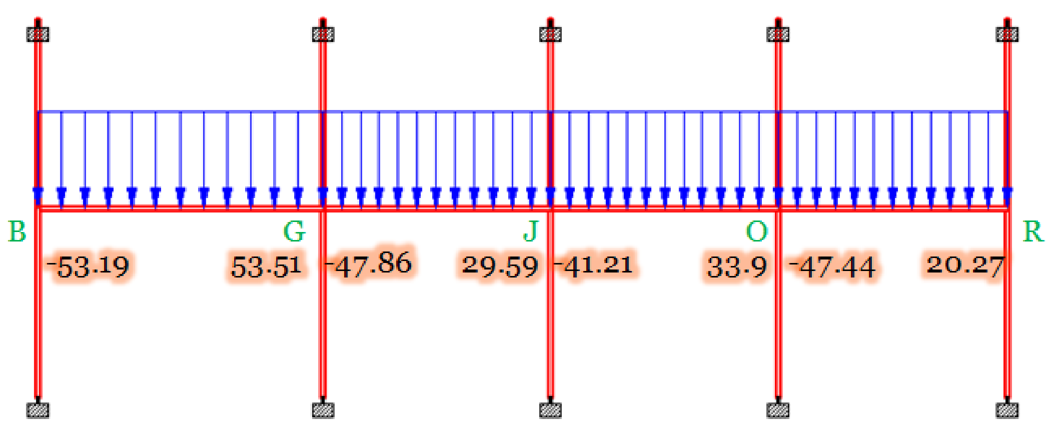
Hence, joint moments at B and R = -12.24 kNm

joint moments at G, J and O = -24.48 kNm

The final calculations are shown in the table,

| JOINTS | B | G | | J | | O | | R |
|---|---|---|---|---|---|---|---|---|
| MEMBERS | BG | GB | GJ | JG | JO | OJ | OR | RO |
| Distribution Factor | 0.25 | 0.19 | 0.23 | 0.23 | 0.23 | 0.23 | 0.23 | 0.29 |
| Moments induced due to wind forces | -12.24 | -12.24 | -12.24 | -12.24 | -12.24 | -12.24 | -12.24 | -12.24 |
| Distribution | 3.06 | 4.65 | 5.63 | 5.63 | 5.63 | 5.63 | 5.63 | 3.54 |
| Carry over | 2.32 | 1.53 | 2.81 | 2.81 | 2.81 | 2.81 | 1.77 | 2.81 |
| Distribution | -0.58 | -0.82 | -0.99 | -1.29 | -1.29 | -1.05 | -1.05 | -0.81 |
| Final moments | -7.44 | -6.88 | -4.79 | -5.09 | -5.09 | -4.85 | -5.89 | -6.7 |

After superimposing these moments with those induced due to vertical loading, we get final end moments as shown below,



| Members | Kani's Method End Moments(kN) | Substitute frame Method End Moments (kN) | Kani's Method Vs S/F Method |
|---|---|---|---|
| **BG** | -54.84 | -53.19 | 3 % |
| **GB** | 56.67 | 53.51 | 5.57 % |
| **GJ** | -43.94 | -47.86 | 8.9 % |
| **JG** | 32.66 | 29.59 | 9.3 % |
| **JO** | -39.93 | -41.21 | 3.2 % |
| **OJ** | 34.68 | 33.9 | 2.2 % |
| **OR** | -46.09 | -47.44 | 2.9% |
| **RO** | 31.6 | 20.27 | 35.8% |

Thus from the above comparisons, we can infer that the substitute frame method of analysis is also equally effective in case of wind loading.

Note- the wind analysis by both the methods has been performed only for one direction of the wind.

| Members | S/F without wind load | S/F with wind load | Variation (%) |
|---|---|---|---|
| **BG** | -45.75 | -53.19 | 16.26 % |
| **GB** | 60.39 | 53.51 | 11.39 % |
| **GJ** | -43.07 | -47.86 | 11.12 % |
| **JG** | 34.68 | 29.59 | 14.67 % |
| **JO** | -36.12 | -41.21 | 14.09 % |
| **OJ** | 38.75 | 33.9 | 12.5 % |
| **OR** | -41.55 | -47.44 | 14.17 % |
| **RO** | 26.97 | 20.27 | 24. 84 % |

The end moments of the 2nd floor (BGJOR), under vertical loading and under both vertical and wind loads has been compared in the above table. It is observed that the percentage variation is less than 25% in all the end moments' variation. And while designing with working stress method, for lateral loads, permissible stresses are increased by 25 % so it can be inferred that there's no need to consider wind load effect while analyzing a frame by substitute frame method.

Under limit state design,

$1.2 \times (20.27) = 24.32$ ------------- (1.2=partial load factor for wind load+dead load+live load)

$1.5 \times (26.97) = 40.45$------------- (1.5 = partial load factor for dead load + live load)

And $40.45 > 24.32$, thus it's witnessed that, even if we analyze without considering wind loads, by substitute frame method, our analysis would be safe.

Thus we can conclude that, in regions with wind pressure= 1.5 $kN/m^2$ , there is no need to consider wind loads while analyzing any frame less than or equal to 14.7m in height by substitute frame method.

# Chapter 5

## CONCLUSION

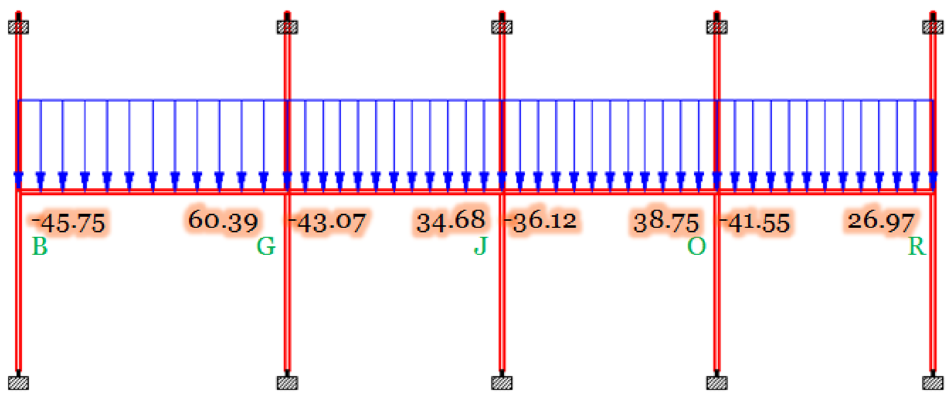
**CONCLUSION**

- Effectiveness of Substitute frame method was checked under wind forces. It was found that substitute frame method can be effectively applied for analysis of frames under wind loads.
- Wind forces can be neglected while performing approximate analysis by Substitute frame method if the building height is 14.7 m or less (in Rourkela region); safe in both working stress method as well as limit state method.

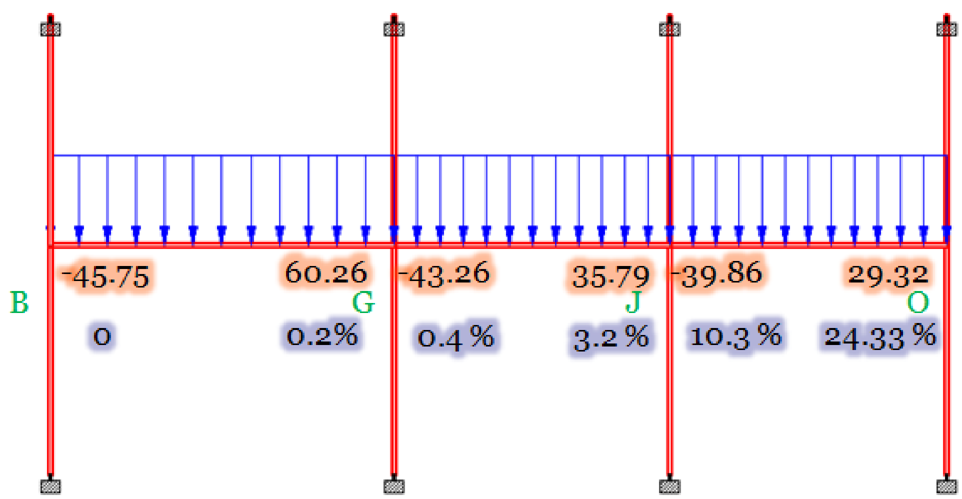
**Future scope of work**

While performing substitute frame method analysis, we can try ignoring some of the far end spans just to reduce the calculations further more. Here is how it affected the end moments of the 1st span when last spans were deducted one by one.

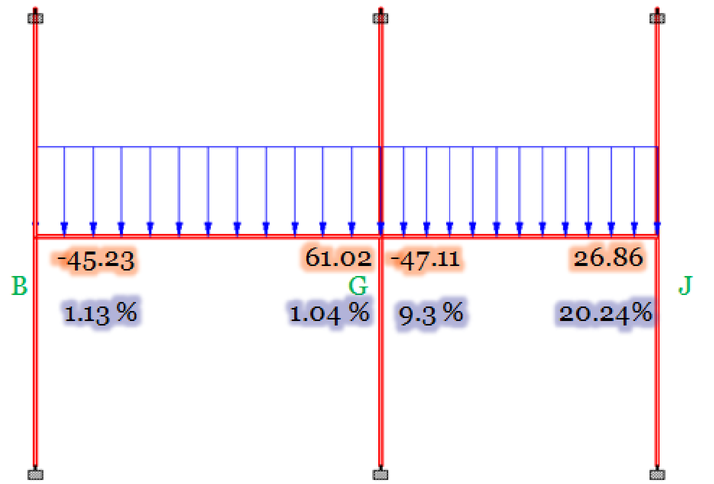
All 4 spans considered-



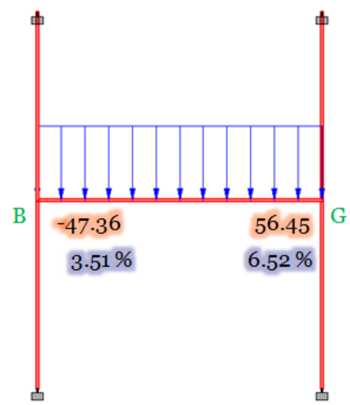

Last 1 span ignored-



Last 2 spans ignored-




Last 3 spans ignored-




The values highlighted in red are the original end moments and those highlighted in violet are the percentage variation in the original end moments.

As we can observe that till the last two spans were ignored there wasn't any significant change in the end moment values of span BG, so if we are to just determine the end moments for span BG, we may neglect last two spans while performing substitute frame analysis, this shall further simplify the method and thus optimize it.

We may go into further detailed optimization, to find out optimum number of spans required to be considered to get desired degree of accuracy in the end moments. This can be applied and checked further for N-storey-N-bay frames.

- The applicability and efficacy of the substitute frame method can be further checked for seismic loading cases also.

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